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BEARING CAPACITY OF PILE FOUNDATIONS CONSIDERING INHERENT ANISOTOROPY OF THE GROUND

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Abstract

In this paper, we examine the applicability of a strain space multiple mechanism model considering inherent anisotropy (Ueda and Iai, 2019) for practical design of pile foundations. In this model, the influence of inherent anisotropy can be expressed by introducing three parameters (a_1, a_2, θ_0). The parameters a_1, a_2 control the degree of anistropy and the third parameter θ_0 expresses the principal direction of inherent anisotropy. The inherent anisotropy is considered here related to the layerd structure of ground, and the bearing capacity of pile foundations might be dependent on the direction of loading as well as the direction of the layered structure of ground.

The model capability is examined as follows. First, we conduct element simulations of cyclic shear tests of sand, in which the effect of inherent anisotropy was experimentally observed, to confirm the validity of the model with given inherent anisotropy parameters. Then, after verifying the applicability of the model for cyclic shear tests, we conduct two-dimensional analyses for a pile foundation. As a result, computed pile loading-settlement relatioship is found to depend on the third parameter θ_0 , which expresses the principal direction of inherent anisotropy. Thus, we show the possibility of the model in the pratical design. However, there is no sufficient observed data of pile foundations influenced by the inherent anisotropy of layered structure ground. In future, we would like to perform experiments of pile foundations in layered ground to experimentally investigate the effect of inherent anisotoropy on the pile behavior and to validate the analytical method.

Keywords: inherent anisotropy, end bearing capacity of pile, strain space multiple mechanism model

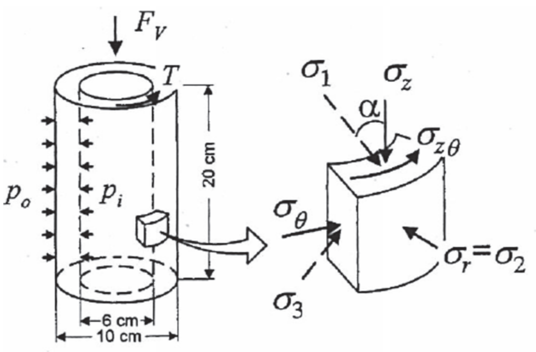
1. Introduction

As soil consists of granular particles, the internal structure (or fabric) is changed by the applied stress due to loading, and the soil behavior becomes anisotropic. This kind of anisotropy caused by stress is called stress-induced anisotropy, whereas the structural anisotropy before loading is called inherent anisotropy.

Many studies have been conducted on the inherent anistropy. Many researchers have conducted hollow cylinder torsional shear tests by changing the principal stress direction α as a parameter (Yoshimine et al. 1998[1], Nakata et al. 1998[2], Yang et al. 2008[3], Symes et al. 1984[4]). Yoshimine et al. 1998[1] conducted torsional shear tests on Toyoura sand using a hollow cylindrical specimen under undrained conditions, and confirmed the effects of the principal stress direction α and the intermediate principal stress coefficient b. The larger the principal stress direction α is, the stronger or more contractive behavior the soil tends to show. In addition, the larger the intermediate principal stress coefficient b is, the stronger or more contractive behavior the soil tends to show. Also, Nakata et al. 1998[2] conducted torsional shear tests on Toyoura sand under undrained conditions using a hollow cylindrical sample, and reported that the larger the principal stress direction α is, the more contractive behavior the soil tends to show.

Symes et al. 1984[4] conducted torsional shear tests with a hollow cylindrical specimen under undrained conditions, and examined the effect of changing the principal stress direction angle α with a constant deviator stress level in addition to the effect of the principal stress direction angles α, which were kept unchanged during monotonic loading. In the latter case (i.e., the principal stress direction α is constant), the larger the applied principal stress direction α was, the more contractive behavior the soil showed and the smaller the initial stiffness and strength were. In the former case, the principal stress direction α was changed from 0° to 45° by keeping the deviator stress level constant during monotonic loading; the effective stress path before and after the principal stress axis rotation agreed with the effective stress path under monotonic loading with a constant principal stress direction α of 0° and 45°, respectively. However, when the principal stress direction α was changed from 45° to 0°, the effective stress path after the principal stress axis rotation did not agree with the effective stress path under monotonic loading with a constant principal stress direction α of 0°.

For simulating this kind of complicated soil responses arising from inherent anisotropy, Ueda and Iai (2019) [5] proposed a constitutive model accounting for inherent anisotropy's influence within a framework of strain space multiple mechanism (Iai et al. 2011[6]).



$$\alpha = \frac{1}{2} tan^{-1}\left(\frac{2\sigma_{z\theta}}{\sigma_\theta - \sigma_z}\right)$$

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} = \frac{1}{2}\left(\frac{\sigma_r - (\sigma_z + \sigma_\theta)/2}{\sqrt{(\sigma_z + \sigma_\theta)^2/4 + \sigma_{z\theta}^2}} + 1\right)$$

Fig. 1 – External forces and stress components on hollow cylindrical specimen (Yoshimine et al. 1998[1])

2. A strain space multiple mechanism model with inherent anisotoropy

The effective stress analysis program FLIP ROSE incorporating the strain space multiple mechanism model (Iai et al., 2011) has been widely used for dynamic analyses of soil-structure systems considering liquefaction during earthquakes. The strain space multiple mechanism model can represent the stress-induced anisotropy of granular materials, while the inherent anisotropy has not been considered.

In the original strain space multiple mechanism model (Iai et al. 2011[6]), the macroscopic effective stress is defined by the tensorial average over contact forces between soil particles, and is found to be composed of the isotropic and deviatoric components as shown in Eq. (1).

$$\boldsymbol{\sigma}' = -p\mathbf{I} + \int_0^{2\pi} q_{\mathrm{Iso}} \langle \mathbf{n} \otimes \mathbf{n} \rangle d\omega \tag{1}$$

where

$$\langle \mathbf{n} \otimes \mathbf{n} \rangle = \begin{bmatrix} \cos\omega & \sin\omega \\ \sin\omega & -\cos\omega \end{bmatrix}$$

For considering the effect of inherent anisotropy, Ueda and Iai (2019) [5] introduced the anisotropic function F into Eq.(1) as follows.

$$\boldsymbol{\sigma}' = -p\mathbf{I} + \int_0^{2\pi} q \langle \mathbf{n} \otimes \mathbf{n} \rangle d\omega \tag{2}$$

where

$$q = F\left(\frac{\omega}{2} - \frac{\omega_0}{2}\right) q_{Iso} + q_{Aniso}$$

$$= \{1 + a_2 cos2(\omega - \omega_0)\} q_{Iso} + q_{Aniso}$$

$$q_{Aniso} = -2a_1 cos(\omega - \omega_0)\overline{E_0}p$$

$$F\left(\frac{\omega}{2} - \frac{\omega_0}{2}\right) = 1 + a_1 cos\omega + a_2 cos2\omega$$

a_1, a_2 : anisotropic parameter

ω_0 : normal-vector direction of the bedding planes relative to the x axis

where q_{Iso} and q_{Aniso} are the isotropic and anisotropic components of the virtual simple shear stress, respectively. Three additional parameters, a_1, a_2, and θ_0 (=ω_0/2), are newly introduced to represent the effect of inherent anisotropy; the third parameter, θ_0, expresses the principal direction of inherent anisotropy (e.g., the normal-vector direction of bedding planes relative to the x axis).

In this study, we investigate the applicability of the extended strain space multiple mechanism model to inherently anisotropic behavior of soil specimens observed in laboratory tests as well as to pile foundation responses such as bearing capacity of pile.

3. The simulation of element test for saturated sand with inherent anisotoropy

Laboratory tests performed by Yoshimine et al. 1998[1] (hereafter called Case 1) and Symes et al. 1984[4] (Case 2) for saturated sand having inherent anisotoropy are simulated to verify the applicability of the strain space multiple mechanism mode accounting for inherent anisotropy. Table 1 and Table 2 show the soil physical/mechanical and dilatancy parameters, respectively. Figure 2 shows the procedure of element test simulation. After conducting isotropic consolidation under stress-controlled and drained conditions, displacement-controlled shear loading in an arbitrary direction was applied under undrained conditions. The principal stress direction angle α can be associated with the direction angle ω of the applied displacement during shearing (i.e., $\omega = 2\alpha$). The inherent anisotropy parameters were specified as follows: a_1 = 0.12, a_2 = 0.20 and ω_0 = 180 for Case 1 and a_1 = 0.20, a_2 = 0.20 and ω_0 = 180 for Case 2.

Table 1 – Parameters for soil

	Case 1	Case 2
Wet density ρ (t/m^3)	1.89	2.00
Shear modulus G_{ma} (kPa) *1	5.85×10^4	8.45×10^4
Bulk modulus K_{ma} (kPa) *1	1.52×10^5	2.20×10^5
Reference confining effective stress σ'ma (kPa) *1	75.0	98.0
Reference parameter m_G, m_K *2	0.5	0.5
Internal friction angle ϕ (°)	42.0	39.7

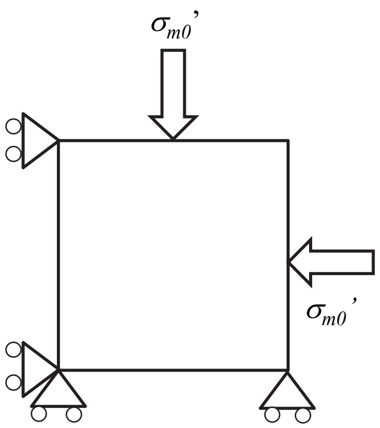
*1 G_{ma} and K_{ma} are shear modulus and bulk modulus at a reference confining effective stress σ'ma

*2 Shear modulus G and bulk modulus K for arbitrary mean effective confining stress σ'm are calculated by the following equations.

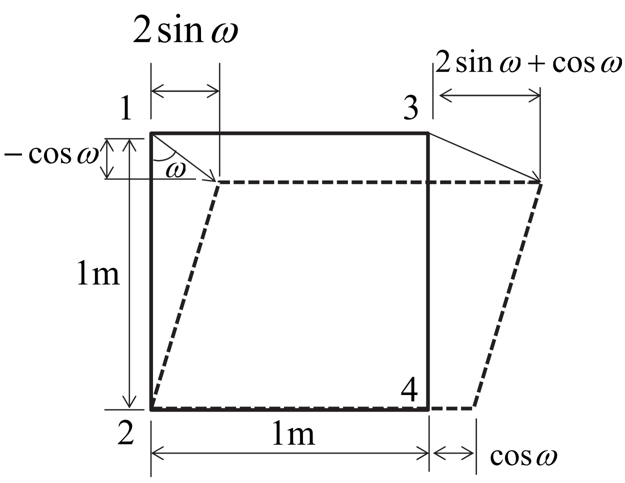
$$G = G_{ma}\left(\sigma_m^{'}/\sigma_{ma}^{'}\right)^{m_G}, K = K_{ma}\left(\sigma_m^{'}/\sigma_{ma}^{'}\right)^{m_K}$$

Table 2 – Dilatancy parameters for soil

Symbol	Parameter designation	Case 1	Case 2
ϕ_p(°)	Phase transformation angle	28.0	28.0
$-\varepsilon_d^{cm}$	limit of contractive component	0.20	0.20
$r_{\varepsilon_d^c}$	Parameter controlling contractive component	3.00	9.00
r_{ε_d}	Parameter controlling dilative and contractive component	0.20	0.12
q_1	Parameter controlling initial phase of contractive component	2.50	3.00
q_2	Parameter controlling final phase of contractive component	0.75	0.95
l_k	Power index of bulk modulus for liquefaction analysis	2.00	2.00
r_k	Reduction factor of bulk modulus for liquefaction analysis	0.50	0.50
S_1	Small positive number to avoid zero confining pressure	0.005	0.005
c_1	Parameter controlling elastic range for contractive component	1.00	1.00



(a) Isotropic consolidation

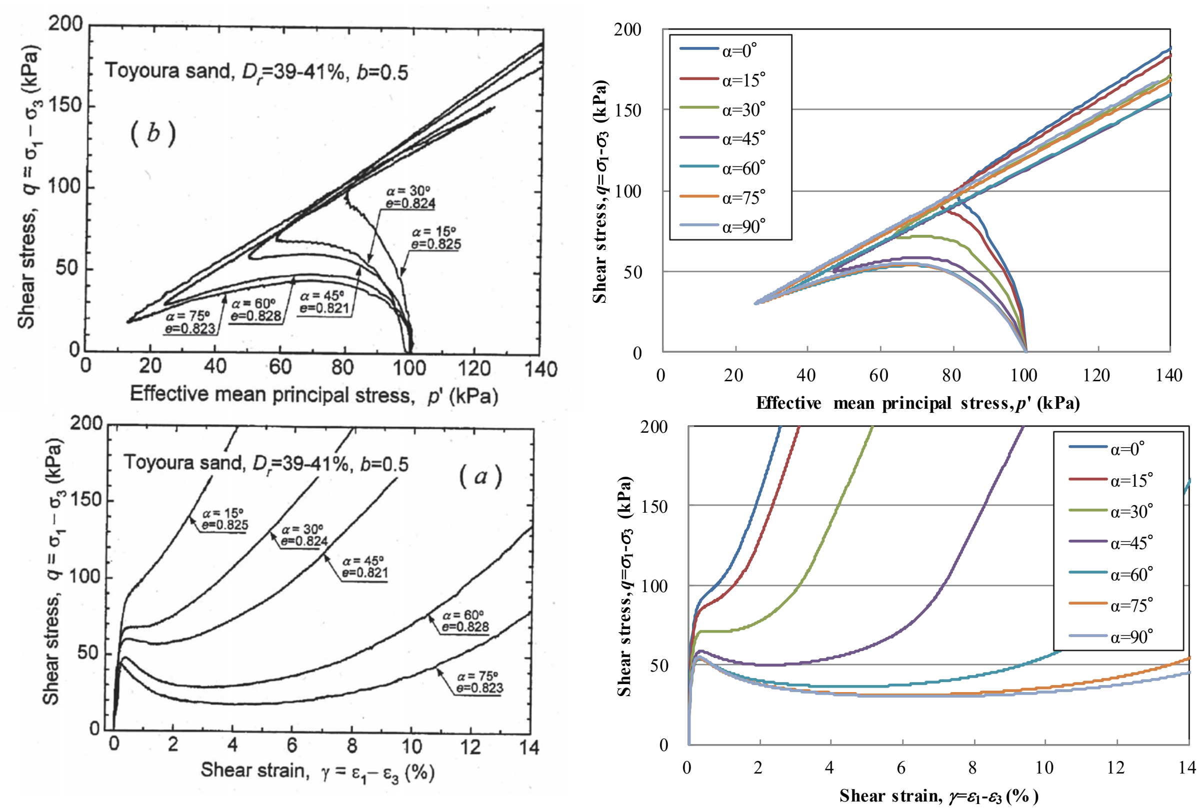


(b) Shear for arbitrary direction (ω=2α)

Fig. 2 – Simulation of element test

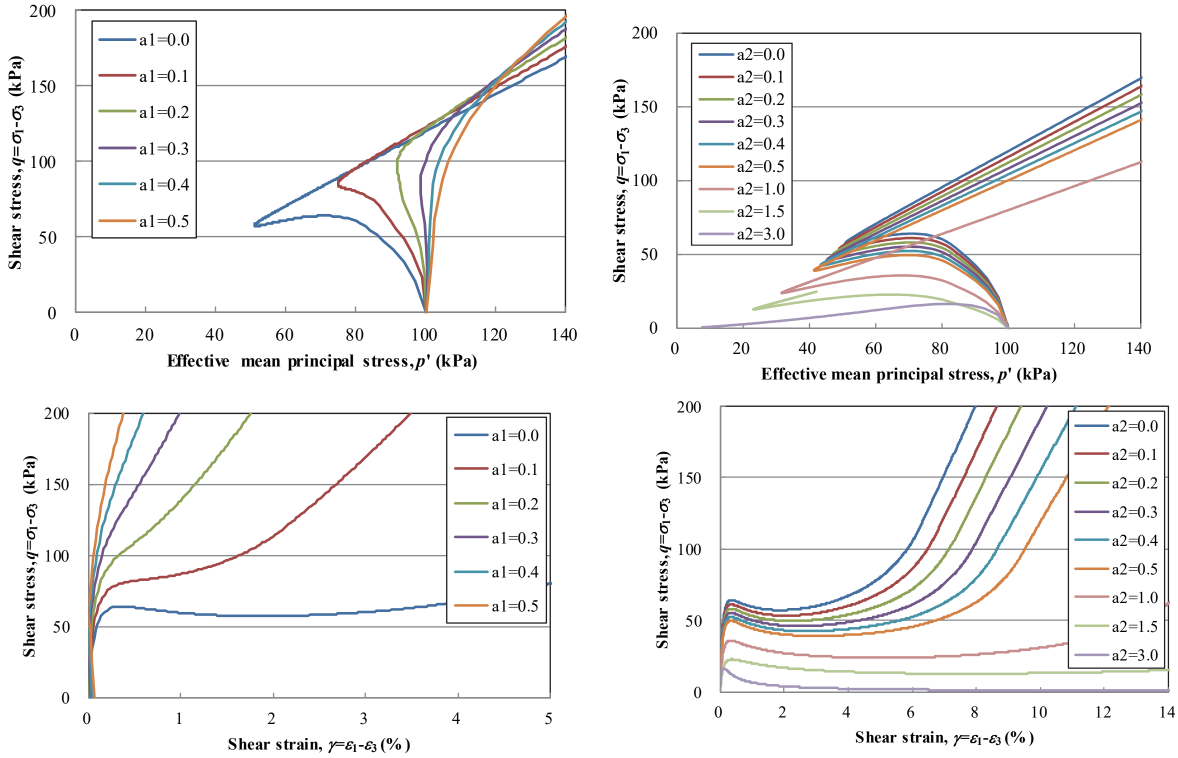
Figure 3 shows the results of the element test simulation (Case 1, shown in Masuda 2016[7]. As the principal stress direction is larger, the behavior becomes more contractive. The simulated stress-strain relationship and effective stress path reasonably capture the dependency of experimental results on the principal stress direction. Figure 4 illustrates the degree of influence of the anisotropy parameters a_1 and a_2 on simulated stress path and stress-strain relationship. As a_1 is larger or a_2 is smaller, the behavior becomes more dilative. By using the anisotropy parameters a_1 and a_2, it is possibile to simulate the behavior of saturated sand having inherent anisotropy.

Figures 5 and 6 show the results of the element test simulation (Case 2). In the three cases where the principal stress direction α was 0° (A0), 24.5° (A2), and 45° (A4), the simulated stress-strain relationship and effective stress path almost agreed with the experimental counterparts. As shown in Fig. 6, when the principal stress direction α is changed from 0° to 45° during loading (R1), the simulated stress path well captures the experimental trend: the stress path during the principal stress rotation was shifted from the effective stress path of 0° (A0) to that of 45° (A4). On the other hand, when the principal stress direction α was changed from 45° to 0° during loading (R2), the analysis was unstable. It implies the re-balance of stress on springs for various direction could make the calculation unstable, and more detailed discussion on the re-balance should be done in future. However, in general, the analysis agrees with the tendency of experiments.



(a) Yoshimine et al. 1998[1]
(b) Analysis results (Masuda 2016[7])

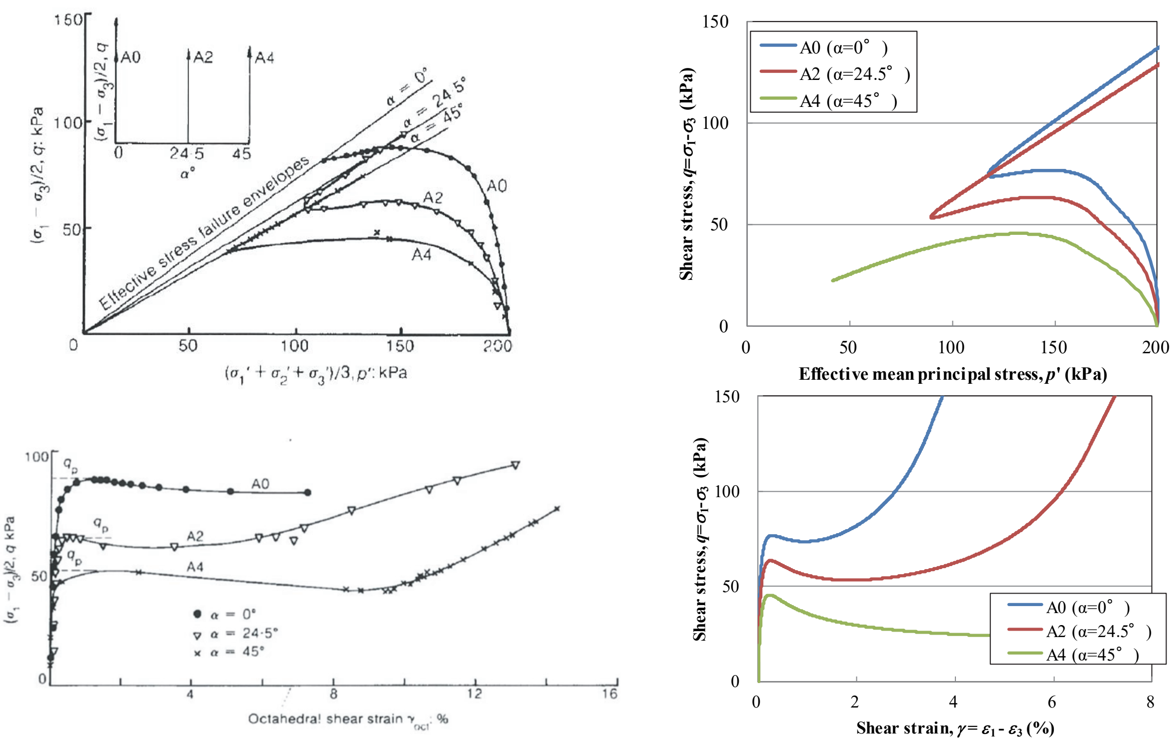
Fig. 3 – The results of element test simulation (Case 1)



(a) Compression shear (α=0° ,a_2=0.0)

(b) Simple shear (α=45°, a_1=0.0)

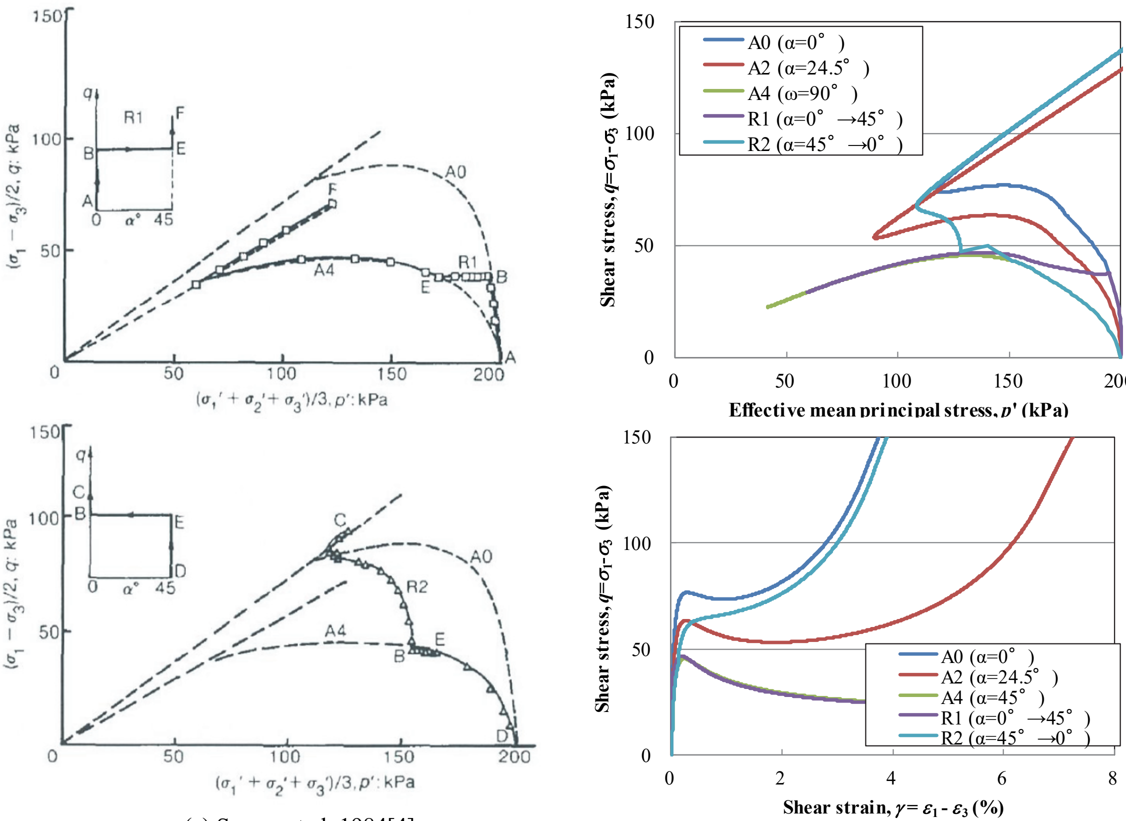
Fig. 4 – The results of element test simulation (Case 1)



(a) Symes et al. 1984[4]

(b) Analysis results (Masuda 2016[7])

Fig. 5 – The results of element test simulation(the principal stress direction α was constant under loading)



(a) Symes et al. 1984[4]

(b) Analysis results

Fig. 6 – The results of element test simulation (the principal stress direction α was changed under loading.)

4. The simulation of pile loading test with saturated sand with inherent anisotoropy

In practice, many designs are based on the results of two-dimensional (2D) analyses. Here, we carry out a series of 2D analyses to simulate the behavior of a vertically loaded pile in inherently anisotropic ground and to examine the applicability of the strain space multiple mechanism model with inherent anisotoropy (Ueda and Iai, 2019[5]). Figure 7 shows the 2D mesh used in the analysis. Table 3 lists the properties of the pile, which was modelled using linear beam elements. Soil parameters are shown in Table 1. The pile was penetrated by applying enforced vertical displacement at the pile head node. Figure 8 illustrates the angle of bedding planes corresponding to the given anisotropic parameter ω_0 between 0° and 180°.

Figure 9 shows the relationship between the pile end resistance and the pile head displacement with consideration of inherent anisotropy's influence. In practice, the pile end resistance at 0.1D is regarded as the bearing capacity. However, numerical stability was not maintained to this level displacement. Thus, we applied vertical displacement of 0.0048 m corresponding to 0.01D. The smaller the anisotropy parameter ω_0 is, the smaller the pile end resistance is. The variation of pile end resistance is 20-30% at maximum. Thus, the consideration of the angle of bedding planes may affect on the bearing capacity of pile.

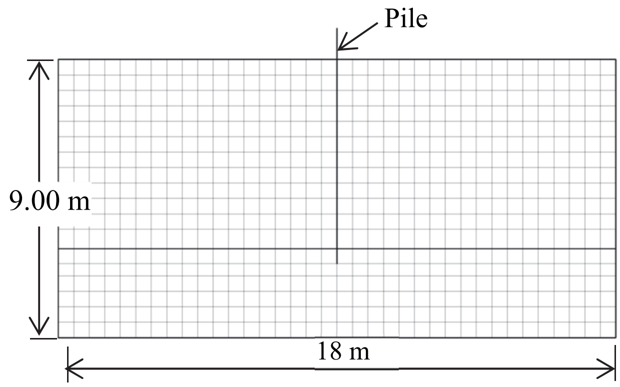


Fig. 7 – 2D mesh model used in the analysis

Table 3 – Parameters for pile

Characteristics	
Diameter (m)	0.48
Thickness (m)	0.032
Young's modulus (kPa)	1.05×10^9
Poisson's ratio	0.35
Area (m^2)	0.045

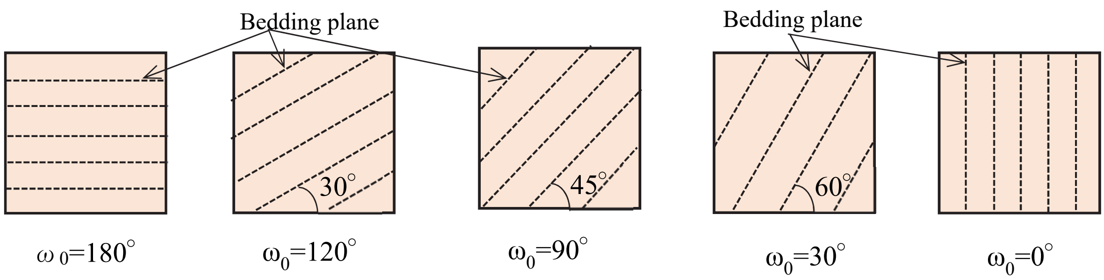


Fig. 8 – Anisotropy parameters

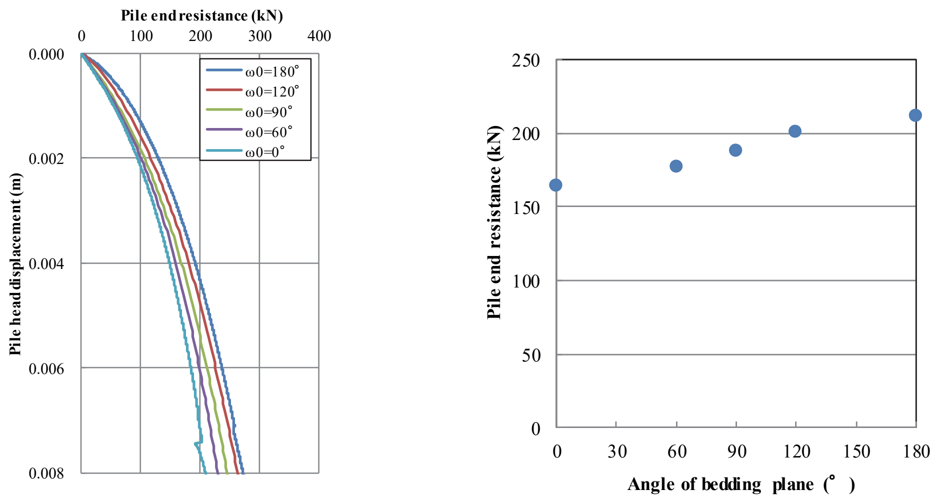


Fig. 9 – Relationship between the pile end resistance and the pile head displacement with inherent anisotropy(Case 1)

5. Conclusions

In this paper, we investigated the applicability of a strain space multiple mechanism model accounting for inherent anisotoropy (Ueda and Iai, 2019[5]) on pile foundation. The following conclusions can be drawn from this study.

(1) Laboratory tests for granular materials under multi-directional monotonic shear loading, in which the principal stress direction angle was kept unchanged in each case, were simulated considering the effect of inherent anisotropy. The constitutive model with a single set of anisotropy parameters was shown to successfully capture the anisotropic soil behaviour (e.g., stress path, stress-strain relationship).
(2) The effect of the principal stress axis rotation with a constant deviator stress level was also numerically studied. It was found that the strain space multiple mechanism model has a capability to simulate the experimental anisotropic behavior during the rotation of the principal stress axis by incorporating the anisotropy parameters.
(3) A series of two-dimensional analyses was performed to simulate the behavior of a vertically loaded pile in inherently anisotropic ground. The simulated results demonstrated that the relationship between the pile end resistance and the pile head displacement depends on the angle of bedding planes; the bearing capacity may decrease 22 % at maximum. Thus, the consideration of the bedding plane angle may be important in practices for the performance-based seismic design of buildings with piles of insufficient length.

However, we did not perform experiments using pile loading test with the inherent anisotropy. Therefore, the range of applicability is limited. In the future, we would like to perform pile loading test with the inherent anisotropy and specify the range of applicability.

6. Acknowledgements

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7. References

[1] Yoshimine M, Ishihara K and Vargas W (1998): Effects of principal stress direction and intermediate principal stress on undrained shear behavior. *Soils & Foundations*, **38** (3), 179-188.

[2] Nakata Y, Hyodo M, Murata H and Yasufuku N (1998): Flow deformation of sands subjected to principal stress rotation. *Soils & Foundations*, **38** (2), 115-128.

[3] Yang Z X, Li X S, Yang J (2008): Quantifying and modelling fabric anisotropy of granular soils. *Geotechnique*, **58** (4), 237-248.

[4] Symes M J P R, Gens A and Hight D W (1984): Undrained anisotropy and principal stress rotation in saturated sand. *Geotechnique*, **34** (1), 11-27.

[5] Ueda K and Iai S (2019): Constitutive modeling of fabric anisotropy in a strain space multiple mechanism model for granular materials. *International Journal for Numerical and Analytical Methods in Geotechnics*, **43** (3), 708-737.

[6] Iai S, Matsunaga Y and Kameoka S (1992): Strain Space Plasticity Model for Cyclic Mobility. *Soils & Foundations*, **38** (2), 1-15.

[7] Masuda S (2016) : Torsional shear test and effective stress analysis on the effect of inherent anisotropy in sand on the undrained shear behavior, Master's Thesis, Department of Civil and Earth Resources Engineering, Graduate School of Engineering.